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ASEISMIC CONSTRUCTION AND DESIGN METHOD FOR MULTI-STOREY COMPOSITE STRUCTURES

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SUMMARY

Using a newly introduced ductile low-rise shear wall with vertical keyways, a seismic resistance design approach for a practical type of composite structure, which consists of a reinforced concrete frame in the bottom floors and masonry structures in the upper floors, has been presented. The purpose of the new design approach is to improve the earthquake resistance of the whole structure by increasing the energy dissipation capacity in the bottom part of the structure. Non-linear analysis shows that, by adopting the newly proposed ductile low-rise shear wall in the bottom of the structure, the lateral deflection of the structure is not much more than that of a structure using conventional solid low-rise shear walls under a small or moderate earthquake excitation, and that even under the attack of a severe earthquake, a stable structural response can be expected for the proposed structure. Thus it is easy for such a structure to achieve the design objective of 'minor damage in a small earthquake and prevention of collapse in a severe earthquake' and the design method is of practical value for similar types of composite structures.

KEY WORDS: aseismic design; low-rise shear wall; keyways; composite structures; seismic response

1. INTRODUCTION

Composite structures with a reinforced concrete frame in the lower storeys and a masonry structure in the upper storeys are widely used in multi-storey buildings in China and some other developing countries. The advantage of this kind of structure is that it can provide relatively large clear spaces in the lower floors desired for some commercial uses, while it can also meet the demands of small housing rooms by using the masonry construction with the intention of reducing the total construction costs of the building. However, a notable problem inevitably arises if such a kind of structure is located in a seismic zone because the structure is usually 'soft' in the bottom storey and stiff in the upper storeys. During an earthquake, the structural deformation will be concentrated in the bottom storey resulting in large lateral displacements of the bottom storey and serious damage or even collapse of the whole structure. To deal with the problem, the present method is to fill some frames in the bottom storey with reinforced concrete shear walls to balance the stiffness between the bottom storey and the upper ones according to the building code for seismic resistance in China, GBJ11-89, which stipulates the detailed required ratio for the stiffness of the bottom storey to that of the upper ones.

The seismic resistance behaviour of those infilled shear walls (or called low-rise shear walls because of small height to span ratio) has been studied quite extensively both in Japan[1,2] and in China recently.[3–6] It has been found that the failure of an ordinary solid reinforcement concrete low-rise shear wall is governed by shear behaviour, with a typical brittle failure mode, and the inelastic energy dissipation capacity is poor. This means that using ordinary reinforced concrete low-rise shear walls to resist a severe earthquake attack can be very dangerous. Therefore, the introduction of ordinary low-rise shear walls to the composite structural

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system cannot eliminate the problem and a new type of wall which can meet the demands on both stiffness and ductility is highly desirable.

The concept of a ductile infilled shear wall, which was originally used in the steel framed high-rise structure, was proposed by Muto in the early seventies.[1,2] Muto found that by introducing numerous vertical slits into the wall panel, the working mechanism of the shear wall can be completely changed, and as a result the ductility and energy dissipation of the shear wall can be greatly improved. This innovative design has already been put into practice in many high-rise buildings in Japan and China. However, it must be noted that the reduction of stiffness and strength of Muto's slit wall is notably large compared with a solid shear wall. The large reduction of stiffness and strength of the slit wall is likely to increase the structural deformation under normal loading conditions (i.e. wind load etc.) and extra structural material is required in order to resist a strong earthquake attack. The actual construction of a slit shear wall is also more complicated than that of an ordinary shear wall. The above-mentioned disadvantages of the slit walls raise a question whether it is suitable for a composite structure where large stiffness and ductility are both strongly required for the shear walls.

To overcome the disadvantages of Muto's slit shear wall, the authors proposed a new ductile design of low-rise shear wall with 'half-through' vertical keyways,[3,4] as shown in Figure 1(a). A layer of concrete with the half thickness of the wall is constructed in the centre of the panel along the keyways and the

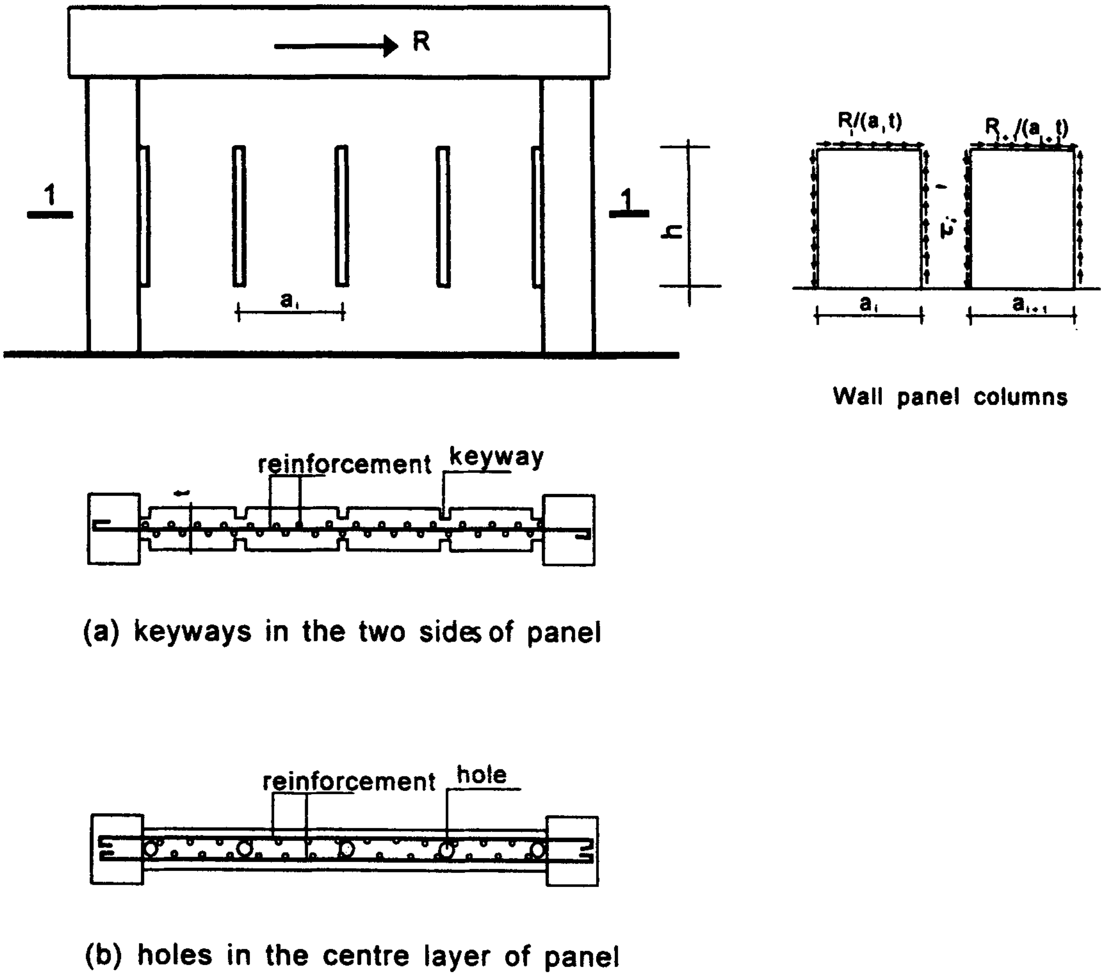


Figure 1. The proposed ductile R.C. low-rise shear wall

reinforcements of the panel are allowed to distribute freely through the keyways. For ease of construction, a weakened part can alternatively be formed by forming vertical holes along the centre-line of the panel and placing the reinforcement in layers outside the holes, as shown in Figure 1(b), for the same design concept. Some experimental studies[7] have shown that the same weakening effect is achieved from the two different methods.

The following features of a shear wall with keyways can be expected:

(1) The decrease of stiffness and strength of the shear wall is relatively small compared to that of Muto's slit wall;
(2) In the elasto-plastic stage, the vertical keyways, acting as fuses, tend to control the developing and extending of cracks so that a diagonal crack cannot develop quickly from the tension zone to the compression zone in contrast to what happens in a solid shear wall. The control of diagonal shear cracks will make the wall endure large deformation;
(3) In the highly-stressed stage, the plastic deformation will be distributed around the keyways in the whole wall panel, which is quite different from a solid shear wall with its plastic deformation concentrated along the main diagonal crack, and the total energy dissipation capacity will be increased;
(4) The post-elastic shear friction between the concrete and the dowel of the reinforcements along the keyways in the wall will change the failure mode of the wall, and increase the damping of the structure. As a result, the earthquake resistance capacity of the structure will be enhanced.

These features of the shear wall with vertical keyways can improve the seismic behaviour of the composite structure during earthquakes.

2. THE BEHAVIOUR OF DUCTILE LOW-RISE SHEAR WALL WITH KEYWAYS

To verify the effectiveness of the proposed shear walls, the authors have conducted cyclic loading tests of eleven low-rise shear wall specimens with vertical keyways, with ratios of the height of keyways to the distance between the keyways h/a varying between 1·33 and 3·33. Tests on three solid shear walls and four slit shear walls have also been carried out at the same time for the purpose of comparison. The size and reinforcement details of those shear wall specimens are shown in Figure 2. The test results show that shear walls with keyways can maintain a very high initial stiffness and yield strength (taken as 85 percent peak strength of the wall), while the brittle failure mode which happens to solid shear walls can be avoided. The test results also show that the shear friction along the keyways between the wall columns (as separated by each keyway in the panel) is most evident at about the peak loading of the shear wall, and that the failure mechanism of the wall columns is very similar to that of flexural failure, resulting in generally good ductility and energy dissipation. Figure 3(a) gives a comparison of nominal shear stress–strain relations for different types of low-rise shear walls obtained from the test, and Figure 3(b) shows the hysteretic loops for the nominal shear stress versus strain for a typical shear wall with keyways. Similar results were also achieved in experiments on fibre-reinforced shear walls with keyways.[5]

Using the non-linear Finite Element Method (FEM), the authors have studied the main factors which influence the behaviour of the shear wall with keyways.[3,4] It is found that the distribution and layout of the keyways are crucial to the overall behaviour of the wall. Parametric studies show that the ratio of the height of the keyways to the distance between keyways h/a should be within a range of 2–4 to ensure sufficient initial stiffness and strength and to achieve good ductility at the same time. A too large value of h/a tends to decrease considerably the stiffness and strength of the wall, while a too small value h/a can lead to a brittle failure mode of the wall. It has also been suggested that a thickness of keyways equal to half the thickness of the wall panel, and the reinforcements across the keyways should be as densely distributed as possible so as to increase the energy dissipation capacity through friction between the concrete and reinforcement along the keyways.

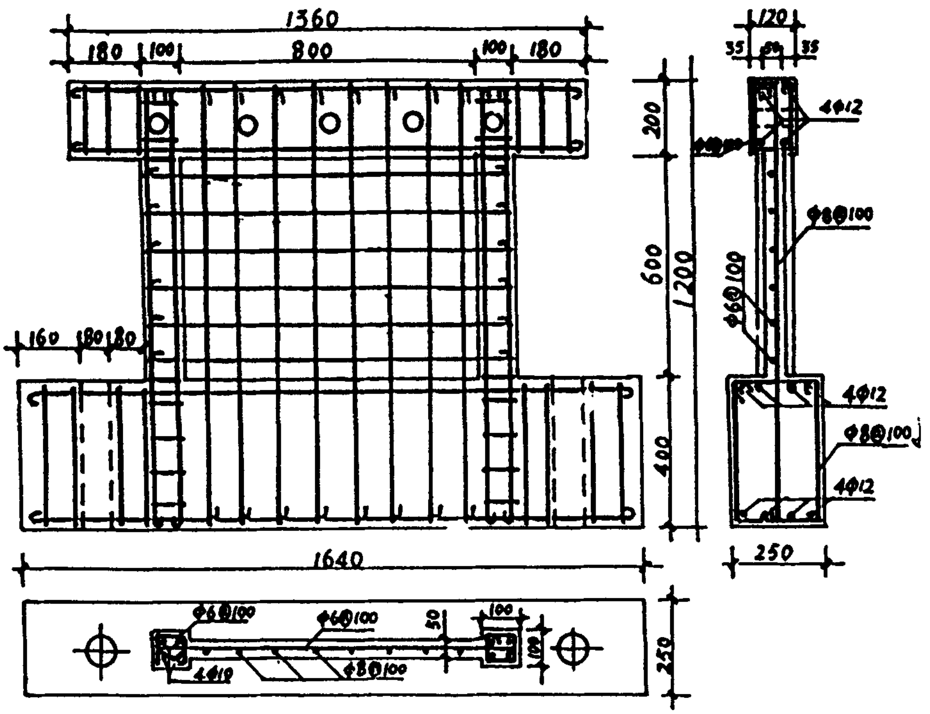


Figure 2. The detail of the tested specimen of low-rise shear walls (units = mm)

3. THE DESIGN AND ANALYSIS OF THE NANJING APARTMENT BLOCK

Brief description of the project

The design method described above was used for a six-storey structure located in Nanjing, China. The bottom two storeys were used as garages and the upper four storeys as apartments. A frame structure is adopted for the bottom two storeys, and masonry wall structure for the upper four storeys. In order to balance the stiffness between the masonry structure and the frame structure, shear walls were used to fill in some column bays at the end of the staircases of the building in the bottom two storeys. The typical floor plans are shown in Figures 4 and 5.

For the reasons stated in the introduction, all shear walls were designed with the proposed ductile innovations. Figure 6 shows the actual detail layout of a six-metre-long-span shear wall in the structures. In the construction the holes in the wall were produced by burying styrofoam bars (with negligible strength) fixed along the reinforcement in the panel, and the reinforcement was placed on both sides of the holes (this method proves to be very easy in the later construction). The actual values of h/a for all shear walls met the general design requirements, and there was no special design requirement for the reinforcement in the wall panel.

Non-linear seismic response analysis

Only the y-direction structure is considered in the analysis (as in Figure 4). Because of the large width to height ratio of the structure (0·73) and the typical shear behaviour of a low-rise shear wall and masonry wall, a shear-type multi-degree-of-freedom model is used in the dynamic analysis,[4] adopting a concentrated mass matrix, that is,

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = -\{r\}[M]\ddot{x}_g \tag{1}$$

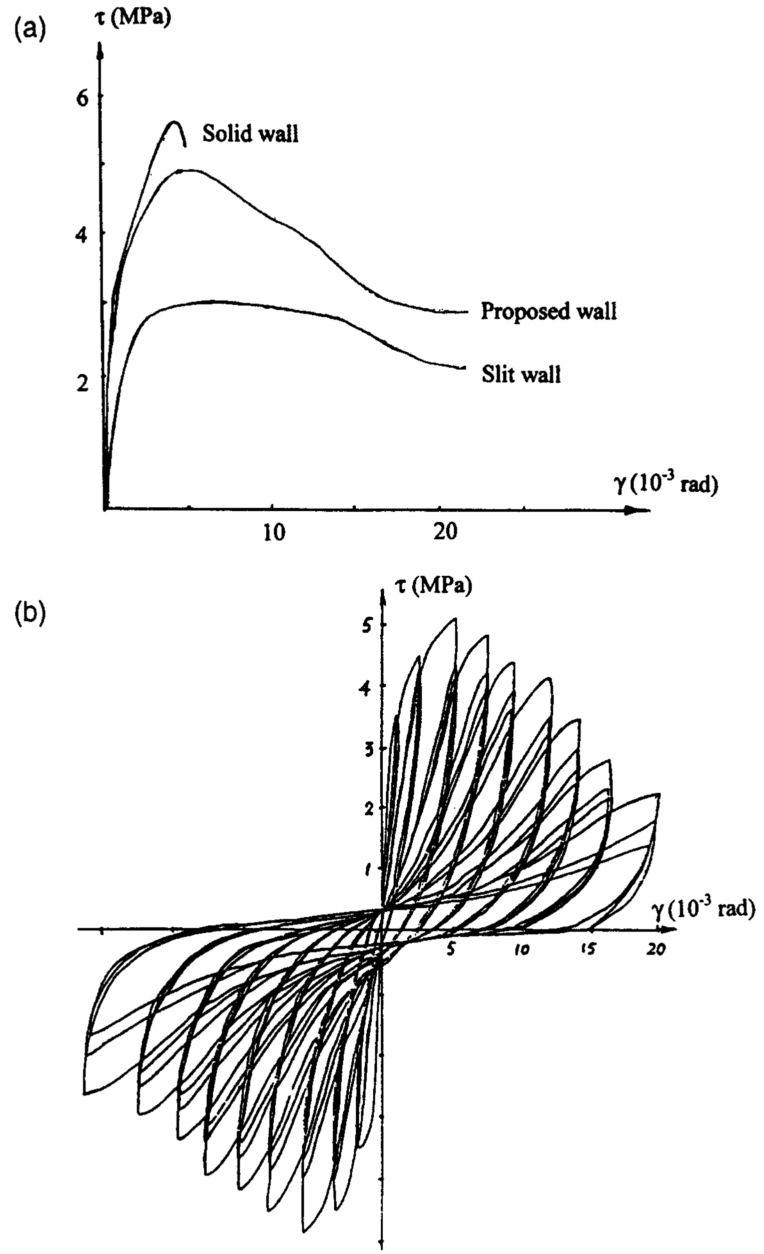


Figure 3. Some test results of the low-rise shear wall: (a) the comparison of skeleton curve of nominal shear stress versus strain relations; (b) the cyclic behavior of the proposed shear wall with keyways

where vectors $\{x\}$, $\{\dot{x}\}$ and $\{\ddot{x}\}$ contain displacements, velocities and accelerations relative to the ground respectively, $[M]$ $[C]$ and $[K]$ are mass, damping and stiffness matrices respectively, $\{r\}$ is the pseudo-static influence vector, and $\ddot{x}_g$ is ground acceleration input.

In constructing the damping matrix, the damping ratios are taken as 0·05 of critical and the Jacobi method is used to solve the characteristic equation. The Newmark-β method is employed to solve equation (1), where

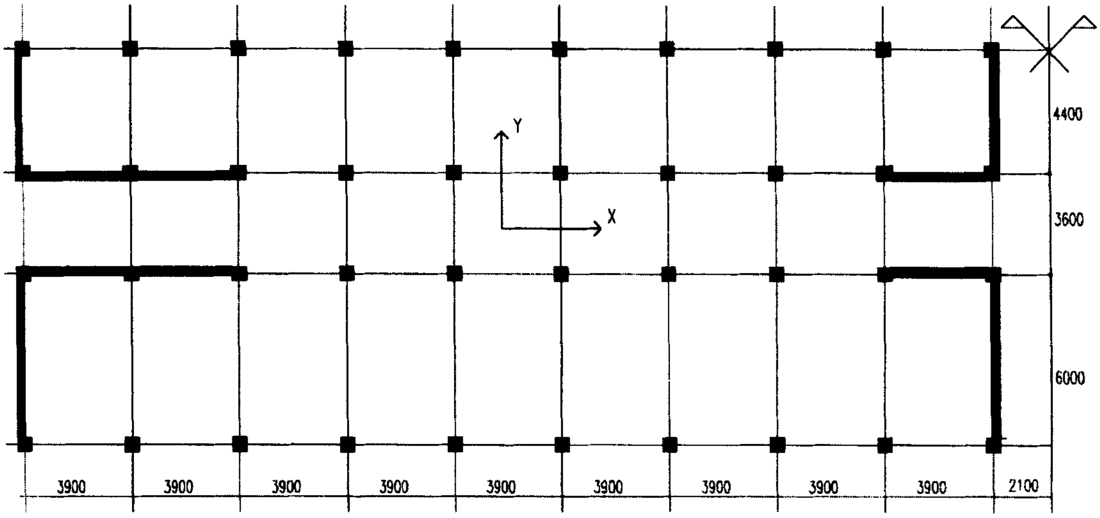


Figure 4. Structural layouts for the first and second floors

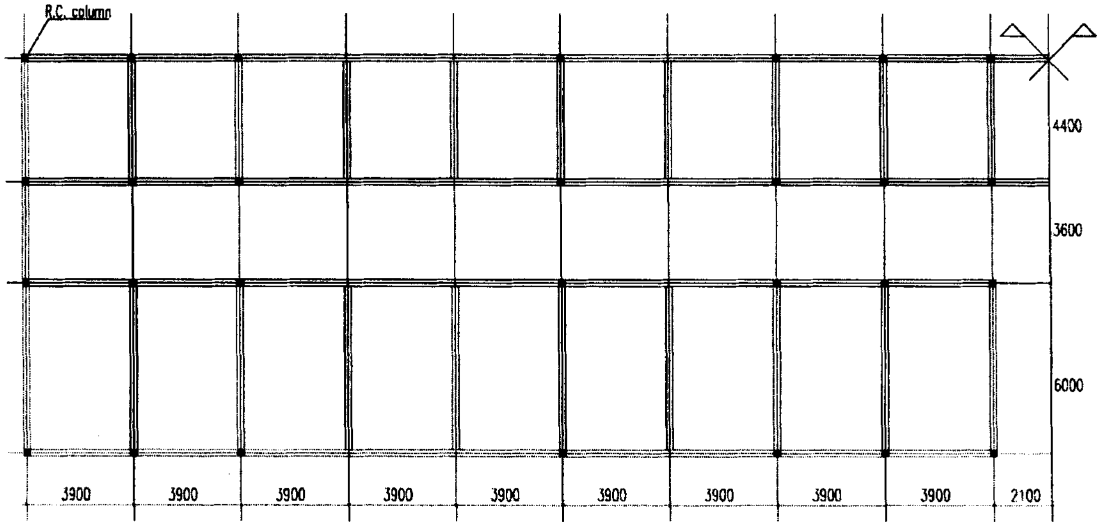


Figure 5. Structural layouts for the third to sixth floors

β is taken as $\frac{1}{4}$ and the minimum time increment is taken as 0·0001 s in order to achieve sufficient accuracy when the structure enters the elasto-plastic stage.

Structural properties for the restoring force models

According to the analysis,[3,4] the restoring force model for the reinforced concrete low-rise shear wall with keyways can be well represented by the model shown in Figure 7(a). For the purpose of practical design work, the simplified formulae for calculating the coefficients for this model are developed based on the following assumptions: (1) the stiffness of the boundary beam of the shear wall is very large; (2) before the shear wall cracks, the shear stress along the vertical keyways can be evaluated by an average shear stress and

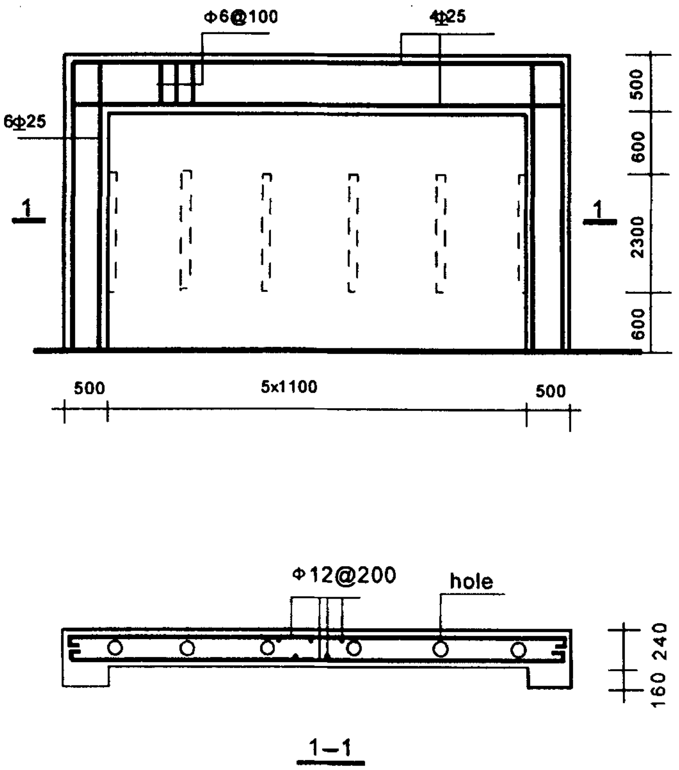


Figure 6. The detail of the adopted ductile wall

(3) when the shear wall reaches its maximum loading, the shear stresses along the keyways reach the ultimate shear strength of concrete.

In the derivation of the formulae for the shear wall before cracking, the wall panel columns (including the boundary columns) are treated independently, as shown in Figure 1. For each wall column, the average shear stress along the keyway is associated with the horizontal shear force acting on the top of the wall column due to the horizontal force applied to the shear wall. The relationships between the horizontal shear forces on the top of the wall columns can then be determined by using the compatibility condition of lateral deflection for each wall column. If the cracking moment of a wall column section is obtained, the relevant horizontal shear force acting on the wall column can be found according to the assumptions made above. After the horizontal shear forces acting on all wall columns are found, the horizontal force at cracking of the shear wall will be the sum of the horizontal shear forces on all wall columns. For the case of the shear wall at maximum horizontal loading, the results of experiments and FEM analysis[3,4] show that: (1) both wall columns and the boundary frame will reach their maximum loading capacities; (2) the failure mode of the wall columns is approximately flexural failure; and (3) the failure of the boundary frame is governed by the compression-shear and/or tension-shear failure of the boundary columns in compression and/or tension. The maximum horizontal load of the shear wall is the sum of the maximum loading capacities of the boundary frame and the wall columns.

In calculating lateral deflection of the shear wall, the shear wall is divided into two parts: the boundary frame and the wall panel. The interaction stresses between the boundary frame and the wall panel are found to be functions of the average shear stress of the wall panel. Then the deflection of the shear wall can be calculated by considering only the boundary frame under the interaction stresses. In the elasto-plastic phase of the shear wall, a reduction of stiffness of the boundary columns is applied to take into account the plastic deformation developed in the boundary frame.

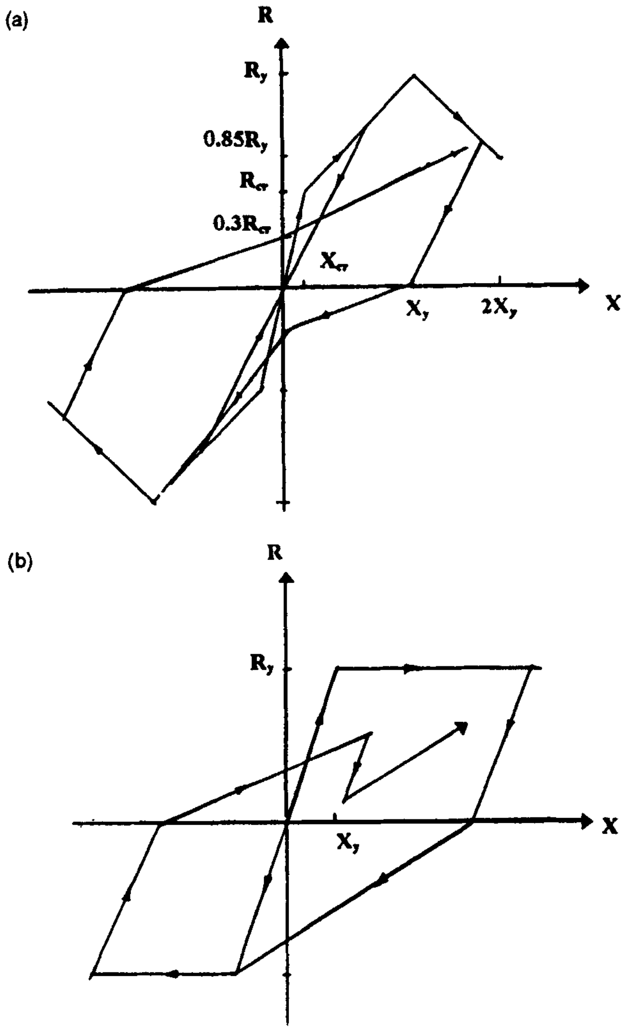


Figure 7. The hysteresis models for calculation: (a) trilinear model for the proposed wall; (b) bilinear model for masonry wall

The derived equations are as follows:

$$R_{cr} = \sum R_i \tag{2}$$

$$R_{i+1} = \frac{R_i \eta_i}{\eta_{i+1}} \tag{3}$$

$$\eta_i = \frac{(1-\xi_i)h^3}{12B_i} + \frac{1{\cdot}5h}{(\mathrm{GA})_i} \tag{4}$$

$$R_i = \frac{2M_{cr,\,i}}{(1-\xi_i)h} \tag{5}$$

$$X_{cr} = \left(\frac{H^3}{24B_c} + \frac{3H}{4\mathrm{GA}_c}\right)R_{c,\,i} \tag{6}$$

where R_i is the shear force resisted by wall column i, h the height of keyways, ξ_i the ratio of thickness of keyways of that of wall panel, and η_i the flexural factor of the wall column i. B_i and $(GA)_i$ stands for, respectively, the bending and shear stiffness of wall column i, H for the height of the shear wall, and $R_{c,i}$ for the shear force resisted by the frame column. B_c and GA_c are, respectively, the bending and shear stiffness of the boundary column.

After the minimum value for cracking moment $M_{cr,i}$ of wall columns is calculated according to the available building code, the corresponding shear can be calculated from equation (5), and then the cracking shear force R_{cr} and displacement X_{cr} of the shear wall with keyways can be estimated from equations (2)–(4) and (6) respectively.

The yield strength and displacement of a shear wall with keyways can be obtained from

$$R_y = \sum R_{y,i} + R_{y,f} \tag{7}$$

$$R_{y,i} = \frac{2M_{y,i}}{h} + 0{\cdot}25a_i t f_c \tag{8}$$

$$R_{y,f} = 0{\cdot}07h_0 f_c + 1{\cdot}5A_s h_0 f_y/s + 0{\cdot}1N_2 \tag{9}$$

$$N_2 = A_{sc} + 2N \tag{10}$$

$$X_y = \left(\frac{H^3}{24B_c^p} + \frac{3H}{4GA_c}\right)R_{y,f} \tag{11}$$

where $R_{y,i}$ and $R_{y,f}$ stand for the shear capacity of wall column i and the boundary column respectively and $M_{y,i}$ is the ultimate bending moment of wall column i, which can also be obtained from the building code. Parameter a_i is the width of wall column i, t the thickness of keyways, f_c the compressive strength of concrete and b and h_0 are, respectively, the width and the effective height of the frame column section. A_s and s are, respectively, the area and the spacing of stirrups in the frame column. N_2 denotes the axial force in the compressive frame column and A_{sc} and f_y denote, respectively, the area and the yield strength of main reinforcement in the frame column. N denotes the external axial loading on the frame column. B_c^p is the bending stiffness of the boundary column in the plastic phase, and $B_c^p = 0{\cdot}8B_c$ may be adopted in a design calculation.

The reliability of the equations (1)–(11) has been verified by many tests and numerical examples.[3,4] Table I gives part of the comparison between the calculated and experimental results, and Figure 8 shows the comparisons of the results from the tests, FEM analysis and theoretical modelling.

For the purpose of comparison, calculation was also carried out by the authors for the same structure but using solid shear walls and Muto's slit shear walls respectively. For the solid shear wall, the restoring model is

Table I. Comparison between the theoretical modelling and the tests

Wall specimen	K^c (10^3 kN/m)	K^c/K^t	R_y^c (kN)	R_y^c/R_y^t	X_y^c (10^{-3} m)	X_y^c/X_y^t
SW1-2	731	1·03	345	0·90	3·15	0·82
SW2-1	234	0·83	215	0·90	3·99	1·14
SW2-3	320	1·15	245	0·94	3·97	1·43
SW2-5	170	0·59	280	1·08	3·99	1·27
SW2-6	277	1·34	195	0·81	3·97	0·95
SW2-7	234	1·14	250	1·09	3·99	1·04
SW2-8	234	0·97	215	0·84	3·99	1·27
SW2-9	234	0·73	185	0·91	3·97	1·04
SW2-11	199	1·08	215	0·84	4·62	1·20

Note: $K = R_{cr}/X_{cr}$; ^{c} calculated results; ^{t} tested results

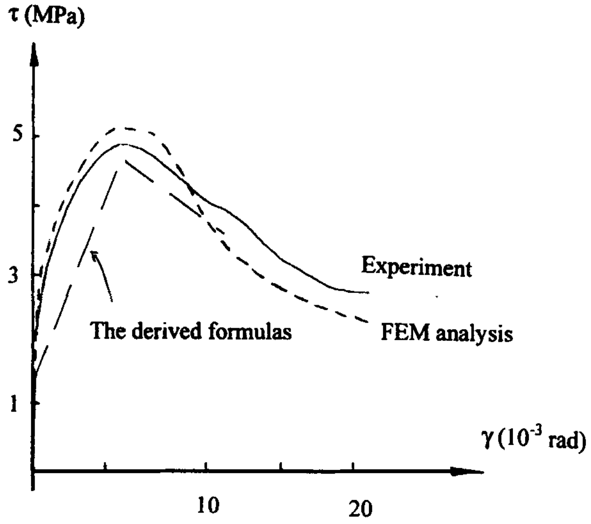


Figure 8. The comparison of the different modeling of the typical wall specimen S

similar to that of a shear wall with keyways, but the coefficients were calculated in a different way.[3] For Muto's wall, the restoring force model was simulated by the trilinear stiffness degrading model but without strength degrading until failure, and the coefficients can be also obtained from equations (2)–(11), setting the thickness of keyway $t = 0$.

For simplicity, the restoring force model for the masonry wall is represented by a stiffness degrading bilinear model, as shown in Figure 7(b). The elastic stiffness and shear resistance capacity can be directly calculated from the corresponding equations in the building code.

Analytical results

From the actual mechanical properties of construction materials, the various structural property values used in the dynamic analyses were calculated, as listed in Table II.

Table II. Structural properties used in the dynamic analyses

Storey No.	1	2	3	4	5	6
Sotrey height (m)	4	4	3	3	3	3
Mass (kg)	142 800	173 500	153 100	153 100	133 200	91 800
Storey elastic stiffness (kN/cm)	98 000 133 000* 37 930†	98 000 133 000* 37 930†	124 640	124 640	104 960	78 720
Storey cracking shear force (kN)	6900 8000* 3300†	6900 8000* 3300†	— — —	— — —	— — —	— — —
Storey yield strength	18 000 20 440* 11 000†	18 000 20 440* 11 000†	23 500	23 500	15 600	9100
Storey yield displacement (cm)	1·572 1·1* 1·57†	1·572 1·1* 1·57†	0·189	0·189	0·148	0·115

Note: * refer to solid shear walls; † refer to Muto's slit shear walls

The El Centro 1940 NS earthquake record and an artificial earthquake accelerogram generated to match the standard response spectrum of the seismic design code in China are shown in Figure 9 and are used as the inputs for the dynamic analysis. The accelerograms were scaled to study the structural response at different response levels. Two sets of analyses (a) and (b) were carried out, as discussed below.

(a) Analysis using the El Centro record. Figures 10 and 11 show the structural responses when the peak acceleration of the wave is equal to 100 gal. and 200 gal. respectively. Table III lists some calculated results.

It can be seen that, under the smaller earthquake excitation, the displacement response of the structure having shear walls with vertical keyways is almost the same as that of the structure with solid shear walls. Under the stronger earthquake excitation, the displacement response of the structure with the proposed walls is larger than that of the structure with solid walls. However, under both levels of earthquake excitation, the displacement response of the structure using the shear walls with keyways is much smaller than that of the same structure with Muto's slit shear walls. In addition, there is not much difference in the maximum base shear forces between three different types of structure when subjected to the smaller ground motion, but with the increase of the peak value of excitations, differences between the responses of the base shear forces become evident. The maximum value of base shear force of the structure using the shear walls with keyways is between that of the structure with solid shear walls and that with Muto's slit shear walls.

Calculations also show that, because of the rapid development of displacement the response of the structure increases rapidly, as a result of yielding in the bottom storey for the structure with either solid shear walls or Muto's slit shear walls when the peak value of earthquake input is 250 gal. It can be interpreted that the two structures suffer severe damage at this stage. At the same level of earthquake excitation, the displacement response is still stable for the structure using shear walls with keyways. As the peak value of the input accelerogram is increased to 300 gal., the maximum storey deflection angle (defined as the storey drift over the storey height) is 1/158, and the maximum top deflection angle (defined as the top displacement over the building height) is 1/432, with the plastic deformation of the structure being still within the ductility limit of the shear walls with keyways.

(b) Analyses using the artificial earthquake record. In order to understand the response of the structures under the attack of a severe earthquake, the peak value of the input wave is adjusted to 400 and 500 gal. respectively. The analysis shows that, for such a strong shaking, the structure with ordinary shear walls fails soon after the yielding of the bottom shear wall. Figure 12, Figure 13 and Table IV present the results of the structures with the proposed walls and with slit walls. The results for the structure with ordinary shear walls are not given here.

It can be seen that, under the excitation of a strong earthquake, the response of the structure with the proposed shear walls, which can survive the attack of above-mentioned earthquake records with peak

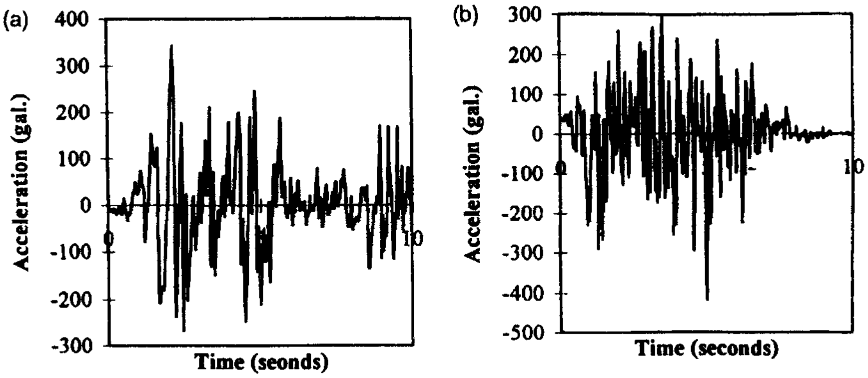


Figure 9. The earthquake record used in the analysis: (a) El Centro accelerogram; (b) artificial accelerogram

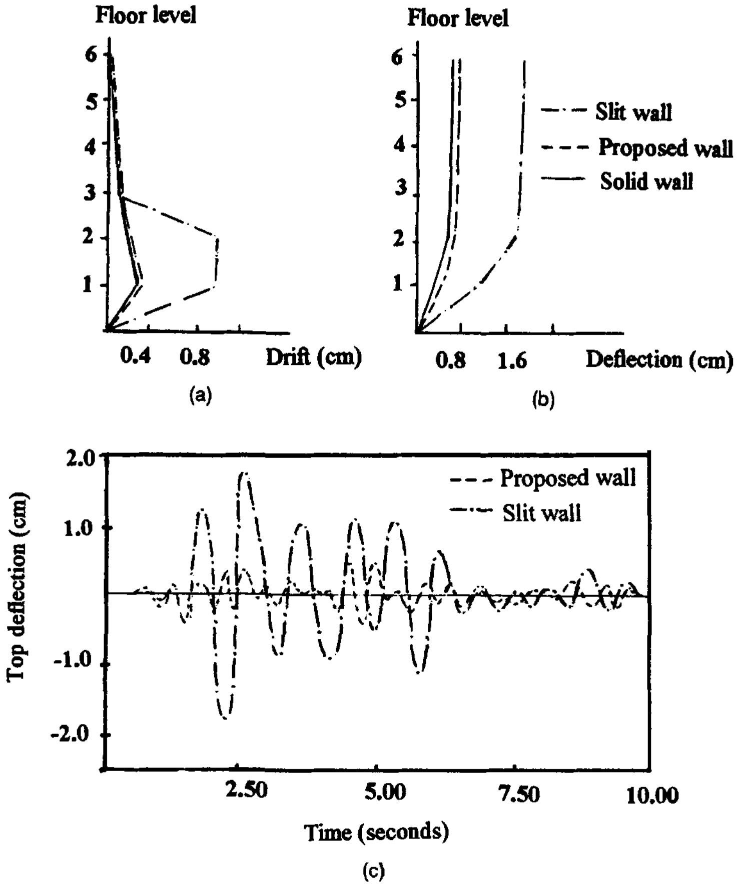


Figure 10. The actual structural response (A_{max} = 100 gal): (a) max. storey drift; (b) max. lateral deflection; (c) top deflection response history

accelerations of 500 gal., is much superior to that of the structure with the slit shear walls. In this case, the ratio of the maximum displacement of the shear wall with keyways to the yield displacement, X_m/X_y is less than 1·6, still within the ductility limit of the wall. For the same level of excitation, X_m/X_y for the structure with slit shear walls is about 3. Although this ductility requirement can be achieved in a slit shear wall, the damage to the structure may be serious for such a level of excitation because of the large inter-storey drift.

4. THE RESPONSES OF THE EXAMPLE STRUCTURES

To support the proposition that the proposed shear wall is a better option for composite structures in general than the other types of shear walls, two additional composite structures are also used here as numerical examples:

(1) *Structure A.* A five-storey structure consists of reinforced concrete frames in the bottom floor and masonry structures in the upper four floors. The typical plan of the bottom floor, which contains shear walls, is shown in Figure 14. Three options for shear walls are taken: ordinary solid shear walls, the proposed shear walls and slit shear walls. For the shear walls with keyways or with slits, six vertical keyways or slits are made with $h/a = 3{\cdot}4$, and the main reinforcement ratios in the wall panel and in the boundary column are 0·008 and 0·009 respectively. Table V gives the typical coefficients of the structure calculated from equations (2)–(11) and the relevant building code;

(2) *Structure B*. Structure B has six storeys with frame structures in the bottom two storeys and masonry structures in the upper four storeys. The storey height is 3·5 m for the bottom two storeys and 3·1 m for the top four storeys. Each of the bottom two floors has a mass of 70 000 kg and the top level has a mass of 80 000 kg. All the other floors have a mass of 102 000 kg each. The plan layout and the other parameters are the same as that of structure A.

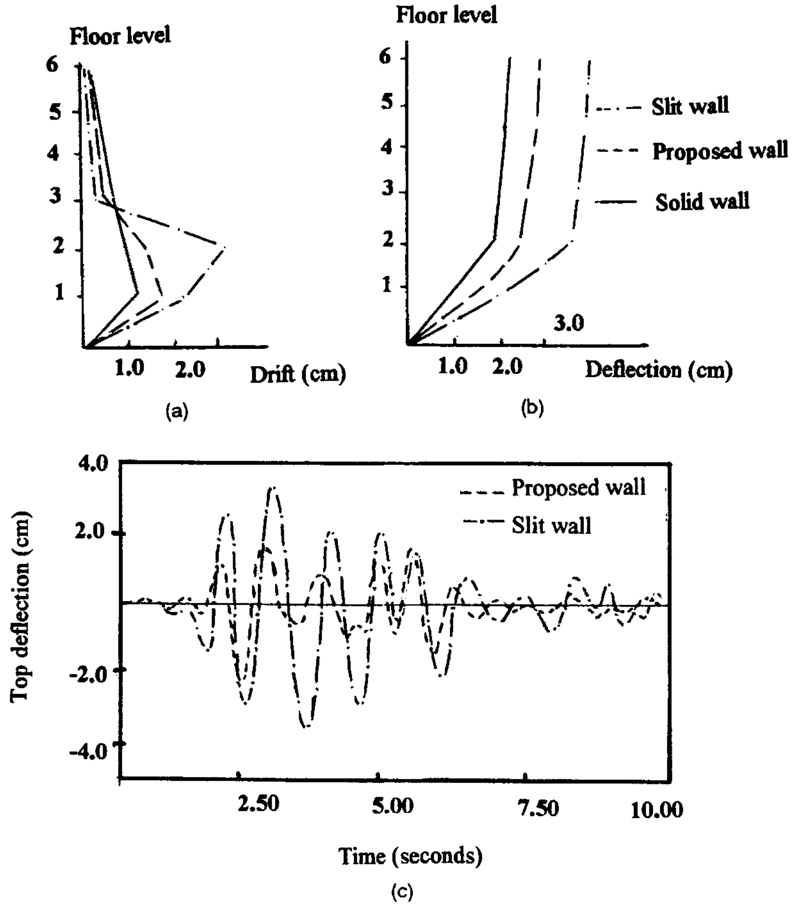


Figure 11. The actual structural response (A_{max} = 200 gal): (a) max. storey drift; (b) max. lateral deflection; (c) top deflection response history

Table III. Structural responses (under El Centro excitation)

Peak value of acceleration	Structural type	Max. storey drift angle	Max. top deflection angle	Max. base shear force (kN)
100 gal.	Wall with keyways	1/1348	1/3163	9904
	Solid wall	1/1582	1/3548	9952
	Muto's slit wall	1/445	1/1065	9320
200 gal.	Wall with keyways	1/292	1/756	20 238
	Solid wall	1/394	1/949	21 320
	Muto's slit wall	1/209	1/546	14 030

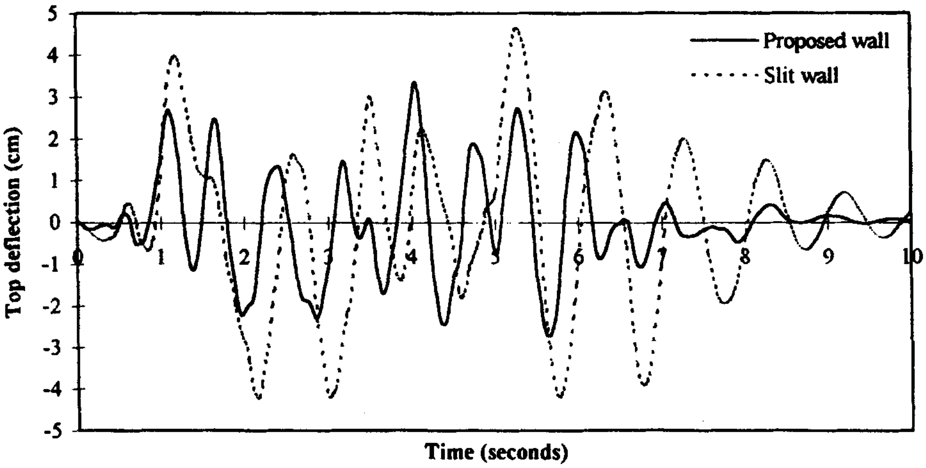


Figure 12. The top deflection response of the structure (artificial accelerogram, A_{max} = 400 gal.)

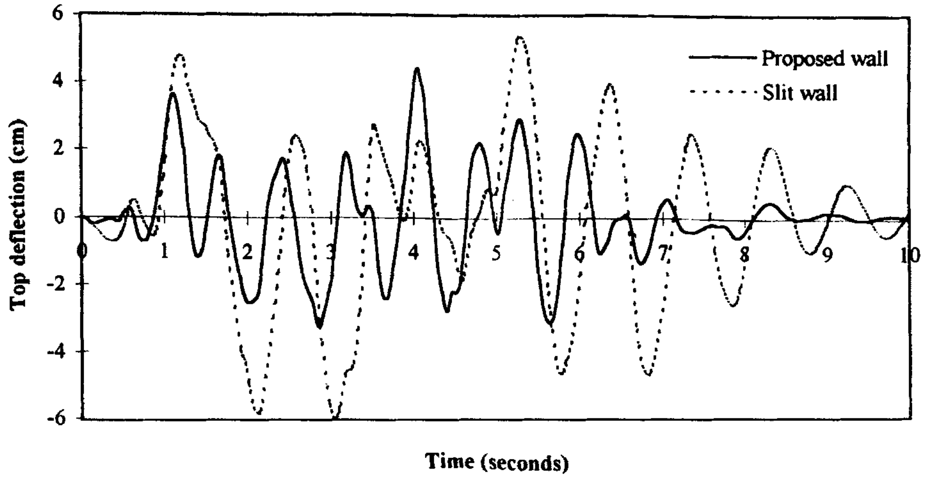


Figure 13. The top deflection response of the structure (artificial accelerogram, A_{max} = 500 gal.)

Table IV. Structural responses (under excitation of artificial accelerogram)

Peak value of acceleration	Structural type	Max. storey drift angle	Max. top deflection angle	Max. base shear force (kN)
400 gal	Wall with keyways	1/235	1/599	20070
	Muto's slit wall	1/125	1/432	13526
500 gal	Wall with keyways	1/158	1/452	22040
	Muto's slit wall	1/98	1/336	14270

The response of structure A

(a) Under the excitation of the El Centro accelerogram. The results are shown in Figures 15, 16 and Table VI. It can be seen that, under the excitation with a peak acceleration of 300 gal., the displacement response of the structure with the proposed walls is larger than that of the structure with ordinary shear

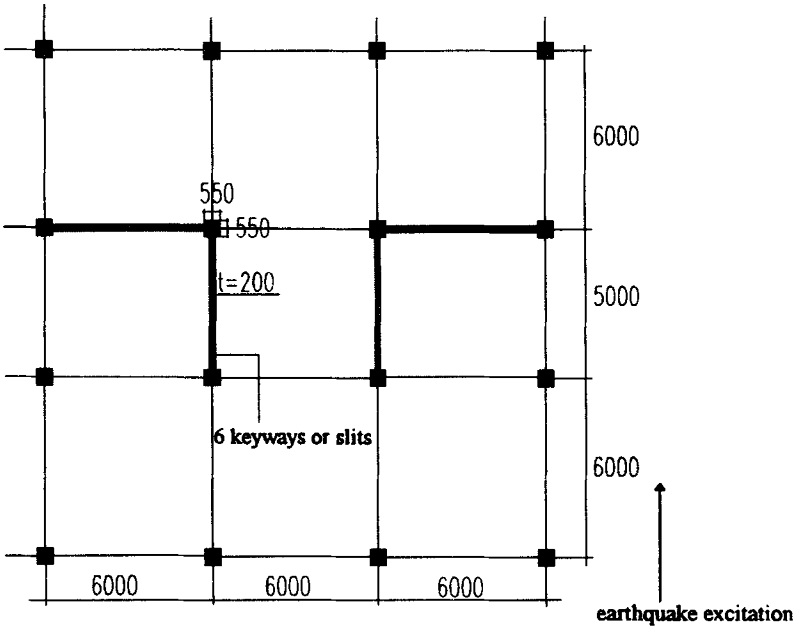


Figure 14. The typical plan of example structures A and B

Table V. Structural properties for the example structure A

Storey No.	1	2	3	4	5
Storey height (m)	3·5	3·1	3·1	3·1	3·1
Mass (kg)	70 000	102 000	102 000	102 000	80 000
Storey elastic stiffness (kN/cm)	44 000 69 000* 21 000†	87 000	87 000	87 000	87 000
Storey cracking shear force (kN)	7100 9000* 3800†	—	—	—	—
Storey yield strength (kN)	21 000 25 000* 15 000†	18 270	18 270	18 270	18 270
Storey yield displacement (cm)	1·30 0·95* 1·27†	0·21	0·21	0·21	0·21

Note: *refer to solid shear walls; † refer to Muto's slit shear walls

walls, but is much smaller than that of the structure with slit walls. When the peak acceleration of excitation increases to 450 gal., the structure with the ordinary shear walls will yield and fail at a very early stage of response. For the same level of excitation (450 gal of peak acceleration), the ratio of maximum displacement to the yield displacement, X_m/X_y, is 6·2 for the slit wall, exceeding its actual ductility limit, and therefore, the structure is expected to fail. For the structure with the proposed shear walls, X_m/X_y is less than 1·9, well within its ductility limit and the structure will not fail at this level of excitation.

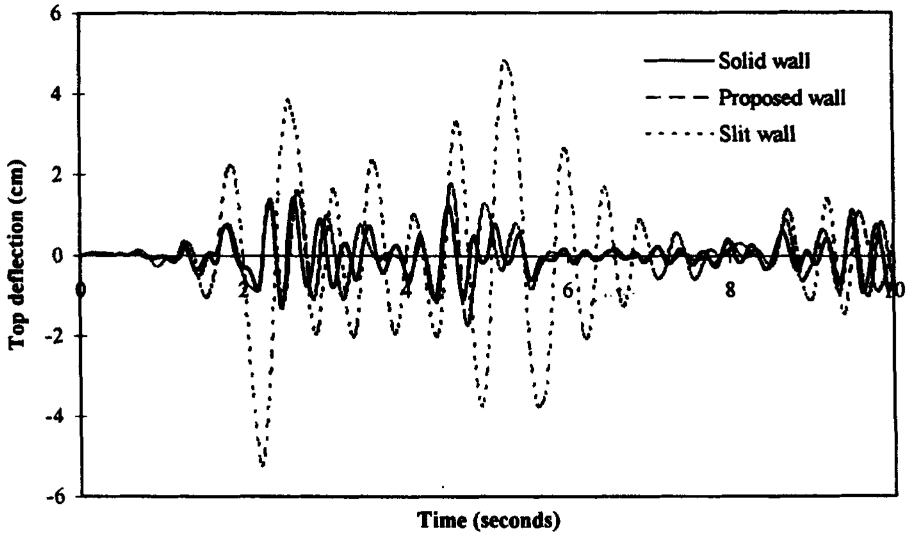


Figure 15. The top deflection response of structure A (El Centro accelerogram A_{max} = 300 gal)

(b) Under the excitation of the artificial accelerogram. The results are given in Figures 17, 18 and Table VII. The responses of the structures under the excitation with A_{max} = 300 gal. are similar to those of the structures under the excitation of the El Centro accelerogram with the same peak acceleration. As to the excitation with a peak acceleration of 450 gal., X_m/X_y for the proposed wall structure and the slit wall structure are 1·8 and 3·0 respectively. Notice that for such an excitation, the structure with ordinary solid walls with fail right after the yielding of the shear walls.

The response of structure B

(a) Under the excitation of the El Centro accelerogram. The results are shown in Figure 19, 20 and Table VIII. Under the excitation with peak acceleration of 200 gal., the displacement response of the structure with the proposed walls is larger than that of the structure with ordinary shear walls, but is much smaller than that of the structure with slit walls. When the peak acceleration increases to 300 gal., the structure with ordinary shear walls yields and fails at a very early stage of its response. For the excitation with a peak acceleration of 300 gal., X_m/X_y of the wall for the slit wall structure is 6·2 and the structure will also fail in this case because a slit shear wall cannot sustain such a high ductility. For the same level of excitation (300 gal.), X_m/X_y for the proposed wall structure is less than 1·7 and the structure will not fail.

(b) Under the excitation of the artificial accelerogram. The results are given in Figure 21 and Table IX. The responses of the structures under the excitation with A_{max} = 300 gal. are similar to those of the structures under the excitation of El Centro accelerogram with A_{max} = 200 gal. When the peak value of the excitation exceeds 400 gal., a rapid increase in deflection due to yielding of the bottom structure occurs for the structure with either solid shear walls or slit walls. This shows that these two structures cannot survive the above-mentioned excitation with a peak acceleration greater than 400 gal. For the proposed wall structure, when the peak value of excitation is increased to 430 gal., X_m/X_y is only 1·69, and therefore a much stronger earthquake resistance capacity can be expected.

5. CONCLUSIONS

In this paper, a practical application of a new design approach is introduced. The approach uses the newly proposed ductile low-rise shear walls with vertical keyways in a composite structure with reinforced concrete

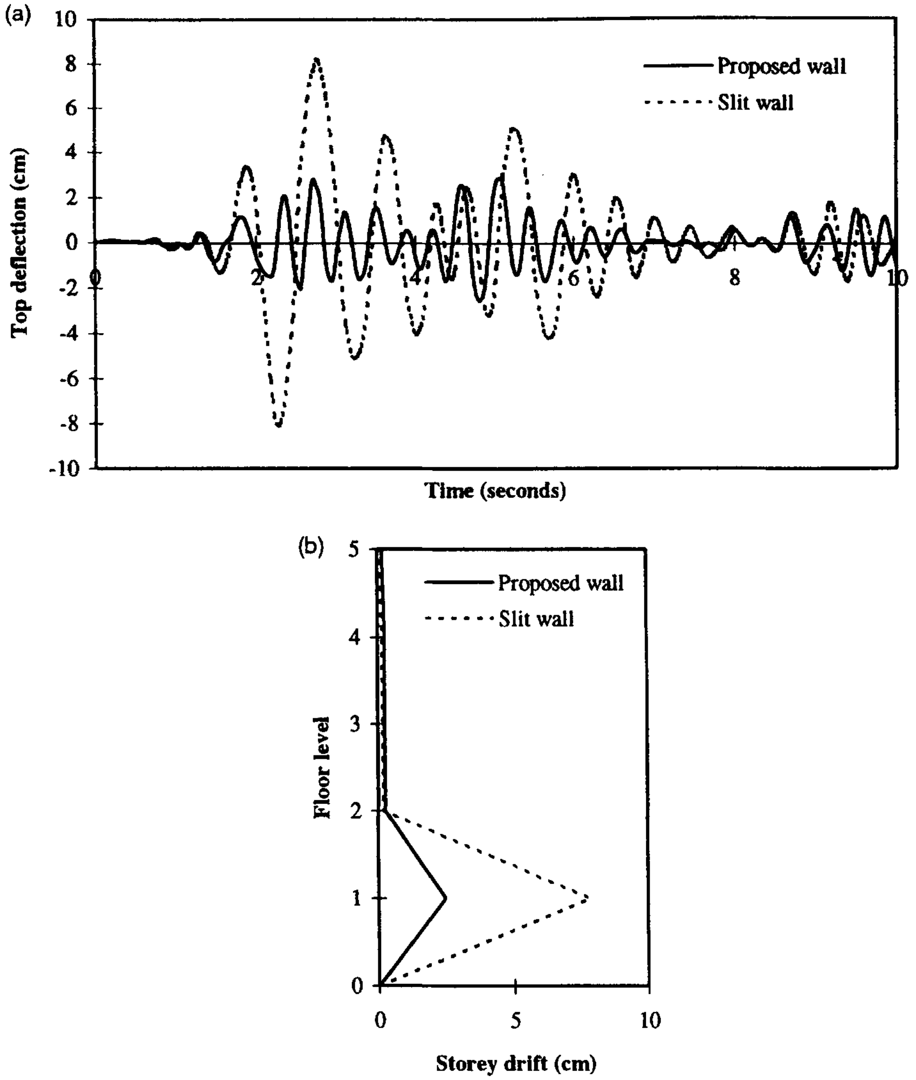


Figure 16. The responses of structure A (El Centro accelerogram, A_{max} = 300 gal.): (a) top deflection response; (b) maximum storey drift

Table VI. The responses of structure A (under El Centro excitation)

Peak value of acceleration	Structural type	Max. storey drift angle	Max. top deflection angle	Max. base shear force (kN)
300 gal.	Wall with keyways	1/293	1/905	21 640
	Solid wall	1/411	1/1126	24 140
	Muto's slit wall	1/76	1/306	16 730
450 gal.	Wall with keyways	1/150	1/558	24 890
	Muto's slit wall	1/47	1/196	17 830

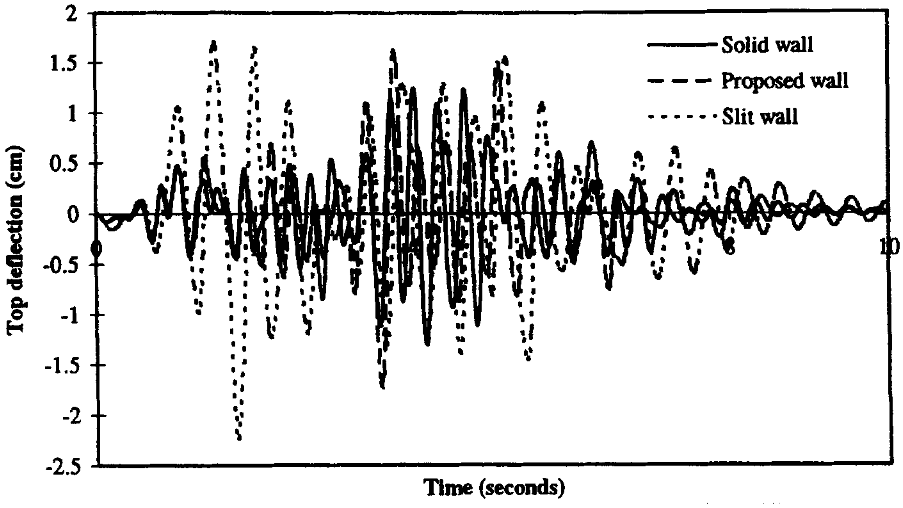
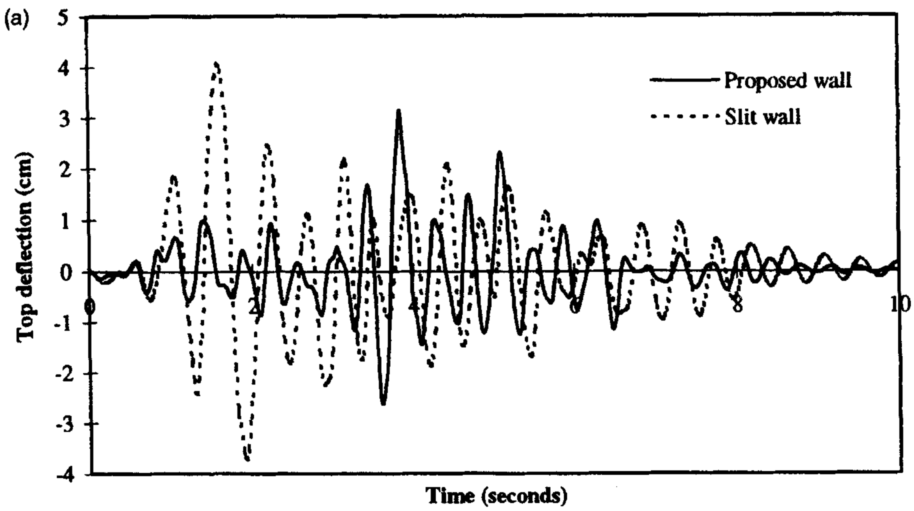


Figure 17. The top deflection response of structure A (Artificial accelerogram, A_{max} = 300 gal.)



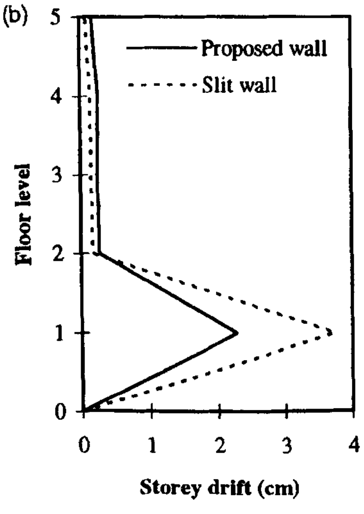


Figure 18. The responses of structure A (Artificial accelerogram, A_{max} = 450 gal.): (a) top deflection response; (b) maximum storey drift

Table VII. The responses of structure A (under excitation of artificial accelerogram)

Peak value of acceleration	Structural type	Max. storey drift angle	Max. top deflection angle	Max. base shear force (kN)
300 gal.	Wall with keyways	1/295	1/921	21430
	Solid wall	1/474	1/1223	22720
	Muto's slit wall	1/201	1/715	14760
450 gal.	Wall with keyways	1/162	1/510	25480
	Muto's slit wall	1/99	1/396	16380

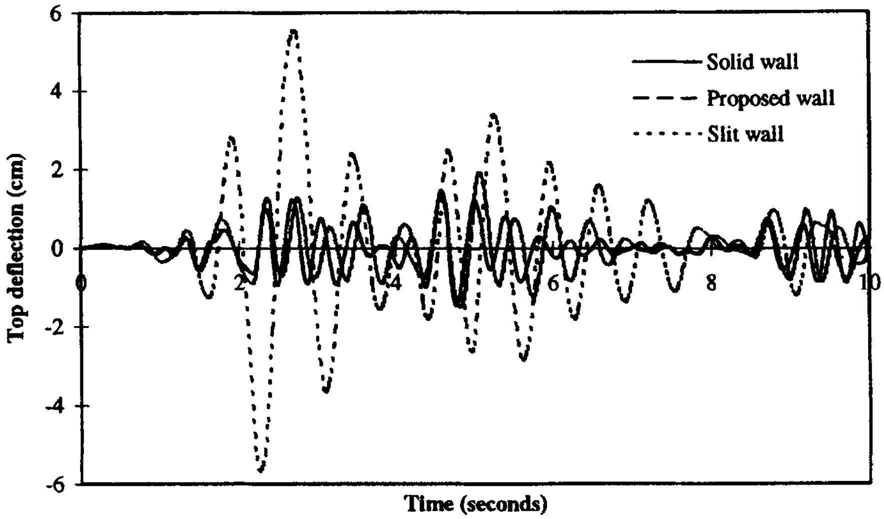


Figure 19. The top response of structure B (El Centro accelerogram, $A_{max} = 200$ gal.)

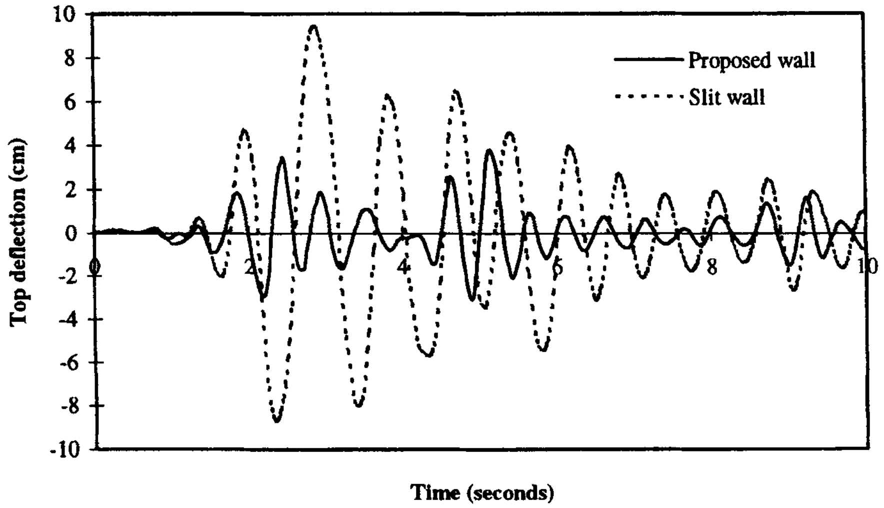


Figure 20. The top deflection response of structure B (El Centro accelerogram, $A_{max} = 300$ gal.)

Table VIII. The responses of structure B (under El Centro excitation)

Peak value of acceleration	Structural type	Max. storey drift angle	Max. top deflection angle	Max. base shear force (kN)
200 gal.	Wall with keyways	1/447	1/1035	16730
	Solid wall	1/638	1/1340	18340
	Muto's slit wall	1/85	1/348	15320
300 gal.	Wall with keyways	1/160	1/522	24870
	Muto's slit wall	1/47	1/210	17790

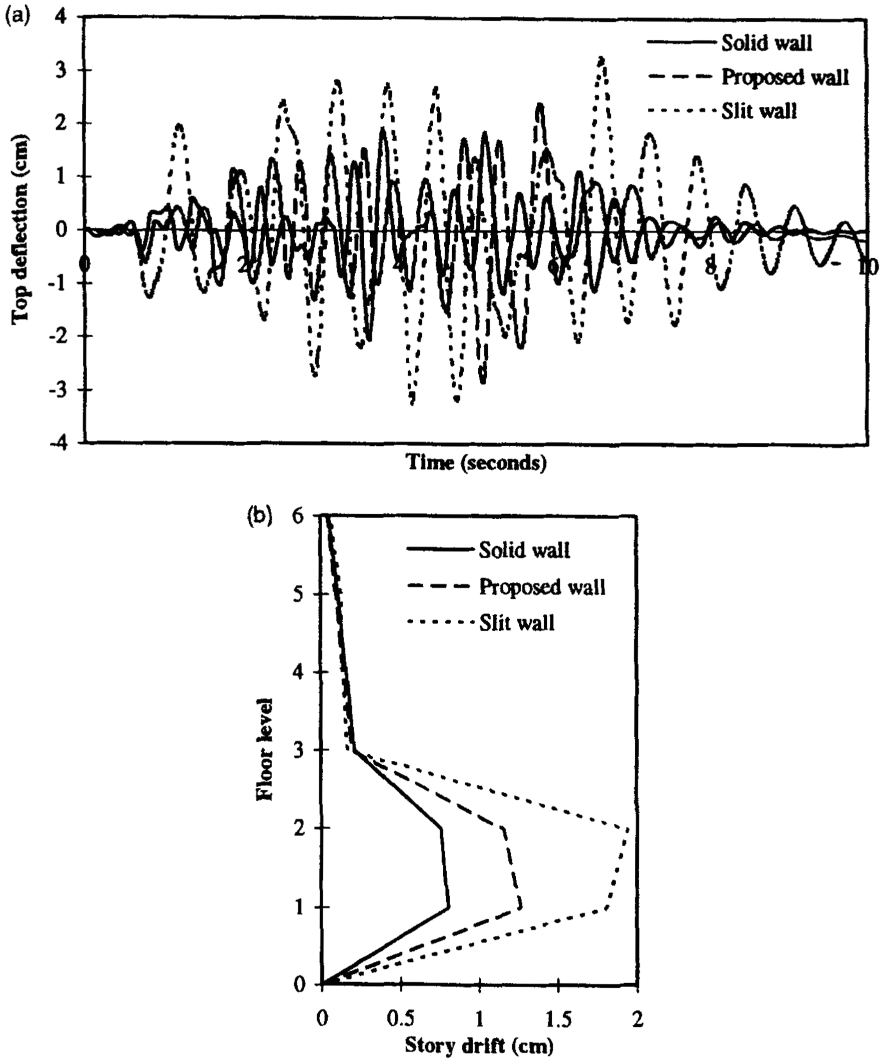


Figure 21. The responses of structure B (Artificial accelerogram, $A_{max} = 300$ gal.): (a) top deflection response; (b) maximum storey drift

frame in the bottom floor and masonry structures in the upper floors. The construction of the shear wall with keyways is very easy, and there is no need for special detailing treatment for reinforcements in the wall panel or in the boundary columns. The improvement of ductility and energy dissipation of the shear wall with keyways is much more significant than that achieved by using ordinary solid shear walls.

Table IX. The responses of structure B (under excitation of artificial accelerogram)

Peak value of acceleration	Structural type	Max. storey drift angle	Max. top deflection angle	Max. base shear force (kN)
300 gal.	Wall with keyways	1/276	1/676	22210
	Solid wall	1/433	1/942	23280
	Muto's slit wall	1/180	1/591	14800
430 gal.	Wall with keyways	1/160	1/587	24070

The analyses and examples demonstrate that the response characters of the newly proposed shear wall are much improved compared with both conventional shear walls and Muto's slit walls. Using the proposed walls, a composite structure can give high initial stiffness at the elastic stage, which leads to a small seismic response of the structure under small-moderate ground motion, and can also maintain a stable seismic response under a strong earthquake attack. The advantages of the composite structure using the shear walls with keyways are shown by comparing its seismic responses with those of a structure using conventional shear walls or ductile slit shear walls.

A wide application of the suggested shear wall with keyways to similar types of composite structures will be beneficial to the reduction of seismic risk and construction costs. However, detailed numerical analysis of the structure must be carried out to evaluate the effectiveness of the actual seismic resistance capacity.

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